

Experimental Validation of Selective Weakening Approach for the Seismic Retrofit of Exterior Beam-Column Joints

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ABSTRACT: The experimental validation of the concept of selective weakening (SW) for seismic retrofit of existing pre-1970s reinforced concrete frames is herein presented. The SW retrofit strategy is to modify the brittle inelastic mechanism to a more ductile mechanism by first weakening selected parts of the structure. Subsequently, the structure can be further upgraded to the desired strength/stiffness/ductility and energy dissipation capacity. Different levels of performance are achievable, from collapse prevention to damage control. For a beam-column (bc) joint, the proposed SW retrofit involves severing the bottom longitudinal reinforcement of the beam, and if required, adding external post-tensioning tendons. In this paper, the experimental implementation of the SW retrofit for poorly detailed exterior bc joint subassemblies is presented. Four 2/3 scaled exterior bc joint subassemblies are used to investigate the feasibility and effectiveness of selective weakening retrofit. Generally, the experimental results confirm previous numerical findings of the viability of SW retrofit to improve seismic performance of existing bc joints. By reducing the shear demand through beam weakening and/or increasing the joint capacity by adding horizontal axial load from external post-tensioning, the local inelastic mechanism is concentrated to a ductile flexural beam hinge, thus achieving the desirable weak-beam strong column/joint global mechanism. Complementing this paper are earlier numerical results of refined FEM 3D models of the exterior bc joint and macro-model of a multi-storey prototype structure.

1 INTRODUCTION

With the introduction of the Building Act 2004 (DBH, 2004) extending the scope of buildings that could be categorised as earthquake-prone, the significant risks associated with substantial damage and global collapse of existing reinforced concrete (rc) moment-resisting frames is legally recognised. Designed prior to the introduction of modern seismic design codes in the mid-1970s, these rc frames generally have inadequate lateral capacity, detailing for ductile behaviour and capacity design considerations; thus they are particularly susceptible to soft-storey collapse or other brittle element failures (NZSEE, 2006). The urgent need for economical and effective seismic retrofit techniques for rc structures is further highlighted in the recent devastating Sichuan Earthquake, China 2008. Experimental testing of beam-column (bc) joint sub-assemblages (Aycardi et al., 1994; Park, 2002) and rc frames (Calvi et al., 2002) have shown that the excessive damage or failure of bc joints, in particular exterior (or corner) joints, can lead to the global collapse of a building or a large portion of the structure. The poor joint behaviour of older construction can be attributed to: the inadequate shear reinforcement in joint, the poor bond properties of plain round bars reinforcement, the deficient anchorage details into the joint and absence of capacity design (Hakuto et al., 1997; Aizhen, 2001).

Various retrofit or seismic rehabilitation schemes have been previously proposed and implemented for bc joints and rc frames (fib, 2003; NZSEE, 2006; ASCE-41, 2007). The majority of the established methods involve either the strengthening of the joint only or both the joint and column in order to induce plastic hinging in the beams. Alternatively, the demand onto the structure can be reduced by supplementary damping or base-isolation. While most retrofit techniques can theoretically achieve a

targeted structural performance, excessive costs, invasiveness and constructability are still the main issues to be solved prior to wider implementation. In this contribution, the experimental validation of a counter-intuitive seismic retrofit strategy, referred as “Selective Weakening”(SW) retrofit, (Pampanin, 2005) for rc exterior bc joint is presented. This paper complements the numerical investigation of the SW retrofit implemented to a prototype 5-storey frame (Kam and Pampanin, 2008).

2 SELECTIVE WEAKENING FOR SEISMIC ‘STRENGTHENING’ / RETROFIT

2.1 Concept of Selective Weakening for Seismic Retrofit

Despite the variety of retrofit strategies and techniques in the toolbox (fib, 2003; NZSEE, 2006; ASCE-41, 2007) available to engineers, it is not uncommon to find global or local strengthening (Figure 1a) as the typical retrofit strategy. While adding obstructive braces or shear walls may seem structurally efficient, without proper engineering judgement, strengthening-only retrofit may generate failures elsewhere within the structural system such as the foundation. The use of composite materials such as fibre-reinforced polymers (FRPs) for jacketing has shown tremendous potential, though the labour intensity and invasiveness of the retrofit techniques might be deterrent to its widespread application. Alternatively, for higher-end building owners, the reduction of seismic demand by the means of supplementary damping (Figure 1b) and/or use of base isolation system (Figure 1c) has been regular practice, as these allows higher performance levels while being less intrusive. Again, the issue of cost and time/space invasiveness of these common techniques has been the reason for its widespread application, particularly in private buildings. The effects of various retrofit strategies on the structural performance are illustrated in Figure 1 within an Acceleration-Displacement Response Spectrum (ADRS) domain, typical of a capacity spectrum method.

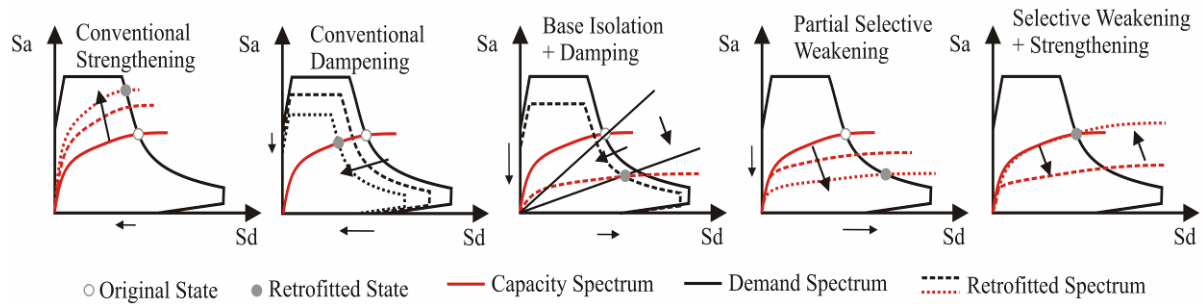


Figure 1: Acceleration-Displacement Response Spectrum (ADRS) illustration of different retrofit philosophies and strategies a) strengthening b) added damping c) base isolation d) partial SW (weakening only) e) full SW (weakening and further enhancement)

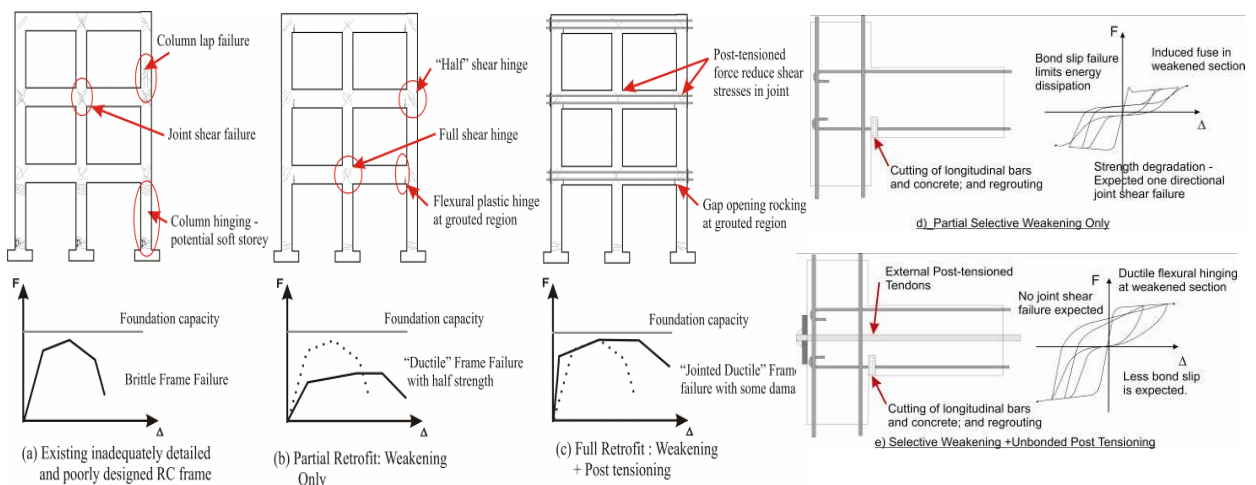


Figure 2: SW retrofit for rc frame: a) existing rc frame b) cutting the bottom longitudinal bars to reduce joint shear stress c) post-tensioning joint and weakened bc interface d-e) Selective weakening on exterior bc joint: and expected force-displacement behaviour – Partial and Full SW retrofit.

Increasingly, retrofit solutions focussed on deformation demand and capacity (e.g. curvature ductility, maximum and residual inter-storey drifts) rather than force/strength, as deformations are considered more effective measures of damage (Pampanin, 2005). In view of such a paradigm shift in the state-of-the-art of seismic retrofit (and design), the proposed selective weakening strategy aims to improve the global inelastic mechanism (deformation capacity) of the structure by first weakening, then upgrading specific/critical structural (or non-structural) elements. Conceptually, where by selectively weakening certain elements and/or re-strengthening the structure, the structure achieves higher deformation capacity with more desirable inelastic mechanisms as illustrated in Figures 1d and 1e. A more illustrative example of the application of SW retrofit for rc frame building is given in Figure 2. By inducing a flexural hinge in the beams by cutting some (or all) longitudinal beam reinforcement at the exterior bc joint face, the overall frame, whilst weakened, becomes more ductile – thus achieving a higher deformation capacity. Further strengthening with external post-tensioning can improve the lateral capacity and energy dissipation while achieving a greater deformation capacity. Figure 2d & 2e provides a comparison on the effect on the expected hysteresis response between partial and full selective weakening retrofit.

2.2 Existing Literature and Previous Research

The idea of reducing the joint demand forces or/and joint-prestressing in order to improve the sub-standard rc bc joint behaviour has been suggested in literature (Priestley et al., 1996). By focusing on increasing the joint shear capacity, researchers in US (Sritharan et al., 1999) and Japan (Hamahara et al., 2007) have investigated the use of joint prestressing/post-tensioning, with mixed results. These researchers were emulating the partially-pre-stressed bc joint presented by Park and Thompson (1977), which formed the basis of considering a contribution of horizontal joint shear capacity being provided by joint prestressing (Clause 15.4.4.2) in the NZ Concrete Standards (NZS3101:2006). As noted that pre-stressing for the retrofit of masonry/heritage structures, inadequate gravity-capacity of beams and columns without sufficient confinement reinforcement are common practice (Pampanin, 2005). In the same publication, the concept of SW retrofit strategy and its possible practical implementation for structural walls, floor diaphragms and rc frames was described. These concepts were subsequently validated with experimental investigations: for the retrofit of shear walls with inadequate shear capacity (Ireland et al., 2007) and for the retrofit of hollowcore floor seating connections (Jensen et al., 2007). ASCE-SEI 41 (2007) standard, outlined the use of external post-tensioning on joint and selective material removal (such as beam weakening) as a valid rehabilitation measure for rc frames. Hitherto, to the authors' knowledge, there is no experimental verification of these retrofit techniques.

2.3 Previous Analytical Study of Selective Weakening Retrofit

The feasibility of using SW retrofit for exterior rc bc joint using detailed finite element models (FEM), using a micro-plane M2 concrete model, MASA (Ozbolt et al., 2001) has been analytically studied (Kam and Pampanin, 2008). The hysteresis behaviour of the as-built and retrofitted bc joints were extrapolated for inelastic time-history analyses of a case-study 5-storey pre-1970s rc frame using Ruaumoko2D (Carr, 2008). The cyclic force-displacement hysteresis, crack and damage pattern computed in the MASA models were in agreement with the experimental response for the as-built specimen (Figure 3). The local-behaviour of full beam weakening (severing 100% bottom longitudinal bars) retrofit was shown to have a positive effect on the displacement capacity of the overall bc joint. The force-displacement behaviour and damage pattern, whilst not being previously validated by experiment, were in agreement with a comparable retrofit solution presented herein (as NS-R1). Two future refinements to the FE model include the improved modelling of a variable axial load and bond-slip cyclic behaviour.

Figure 4 presents the envelopes of the maximum responses from the non-linear time history analyses of a pre-1970 designed rc frame. As expected, the as-built frame has limited energy dissipation capacity with shear failure occurring within the bc joints. Joint rotation is the predominant inelastic mechanism. Inter-storey drift was in excess of 3.5% on average. The SW retrofit frame with weakened-beams (positive flexural capacity) clearly shows a remarkable reduction to the inter-storey drift envelopes. The predominant inelastic mechanism, beam flexural hinging, has more ductility and energy dissipation capacity. When considering the individual elements, the as-built frame would have

likely collapsed as the rotation and curvature demands on the joints and columns respectively were all exceeding the typical collapse limit states.

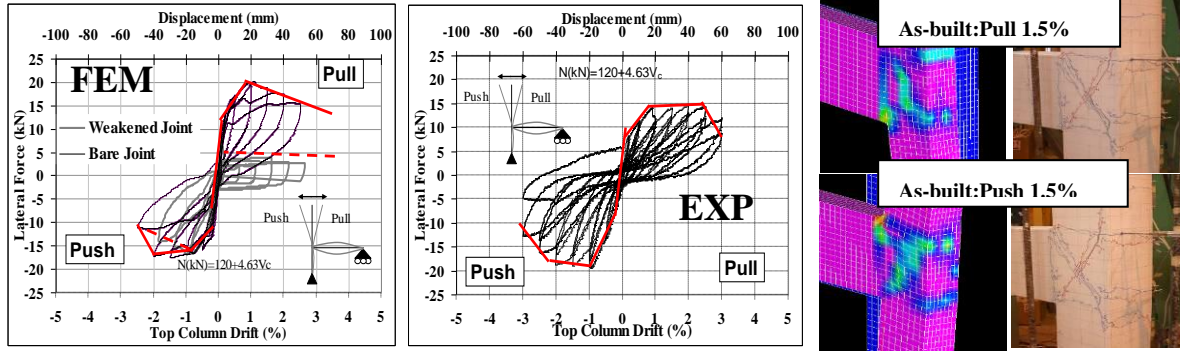


Figure 3: a) As-built and weakened bc joint lateral force - column drift curves– (left) numerical result (MASA); (right) experimental result b) Predicted and observed failure mode and cracking pattern of existing bc joint. Lighter colours on the FEM output are indications of higher strains and stresses.

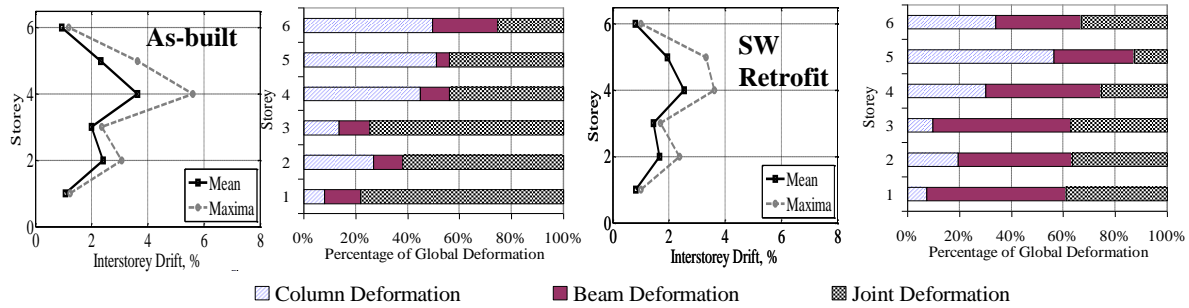


Figure 4: Average of peak inter-storey drift envelopes responses and average global deformation components of the existing and retrofitted frames.

3 EXPERIMENTAL INVESTIGATION

3.1 Specimen Details / Test Matrix

For brevity, only brief description of the experimental program is provided here. The as-built benchmark bc joint, NS-O1 was designed to represent worst typical case pre-1970s construction practice while meeting the requirements of NZS-95(1955). The subassembly is assumed to be located between points of contraflexure, occurring at mid-height of columns and mid-span of the beam, within a 3-bay 3-storey rc frame. The joint has no transverse reinforcement and the beams longitudinal reinforcement are anchored using 180 deg. standard hooks, as shown in 5a. All bc joint units have 230mm x 230mm columns and 330mm deep x 230mm wide beams. Geometry and reinforcement details of the as-built benchmark bc joint is shown in Figure 5a. Standard steel products are used: mild steel and pre-stressing 7-wire tendon yield strength of 330MPa and 1560MPa respectively.

The description of the test units are given in Table 1, outlining the differences between the alternative retrofit solutions. Test unit NS-R1 represents a Partial SW retrofit, where 50% of the bottom longitudinal beam bars are cut. This is done in the lab using a metal grinder (Figure 5b) while for larger specimens, diamond cutters are commercially available. The concrete gap is later re-grouted with SIKATM GP Grout. Test unit NS-R2 is to investigate the effect of external pre-stressing on the poorly detailed bc joint. Test unit NS-R3 is an example of the Full SW retrofit, where the beams were selectively weakened in conjunction with external pre-stressing of the bc joint. The 20mm anchorage plate, anchored with 2 FisherTM 10mm FAZ II anchors, was designed such that a rigid anchorage was achieved. It is expected that commercial pre-stressed anchorage (e.g. VSL, BBR) can be used for practical applications. Only a relatively low pre-stressing force is required for successful joint retrofit, and from laboratory experience, this post-tensioning operation is not very labour-intensive (Figure 5c).

Table 1: Description of Beam-Column Joint Test Units

Test Unit	Description	Beam Bottom Reinforcement s	PT Force (kN)	Concrete Strength, f_c (MPa) ¹	$M_{beam-cal}^2 / M_{column-cal}^3$	$M_{beam-cal}^2$ (kNm)	$M_{joint-cal}^4$ (kNm)
NS-O1	as-built benchmark specimen	4-R10	-	17.5	+1.79 -0.98	± 33.2	+10.4 - 15.5
NS-R1	retrofitted - 50% beam weakening only	2-R10	-	25.6	+0.82 -0.87	+15.1 -29.7	+10.4 - 15.5
NS-R2	retrofitted - 120kN PT only	4-R10	120	28.2	+2.56 -1.39	± 47.4	+12.5 - 21.2
NS-R3	retrofitted - 50% beam weakening + 40kN PT	2-R10	40	24.3	+1.26 -1.07	+23.4 -36.3	+15.4 - 31

Abbreviation: NS=no column lap-splice; O=as-built; R=retrofitted; PT=post-tensioning; R10 = plain round bars with diameter 10mm.

¹ Concrete strength at the day of testing; ² Calculated nominal beam flexural capacity based on concrete compression strain, $\epsilon_c = 0.003$

³ Calculated column flexural capacity at expected varying axial load ⁴ Calculated joint shear capacity based on principal tensile stresses (e.g. Priestley et al, 1996).

Positive moment corresponded to the Pull direction, in which the bottom of the beam are in tension.

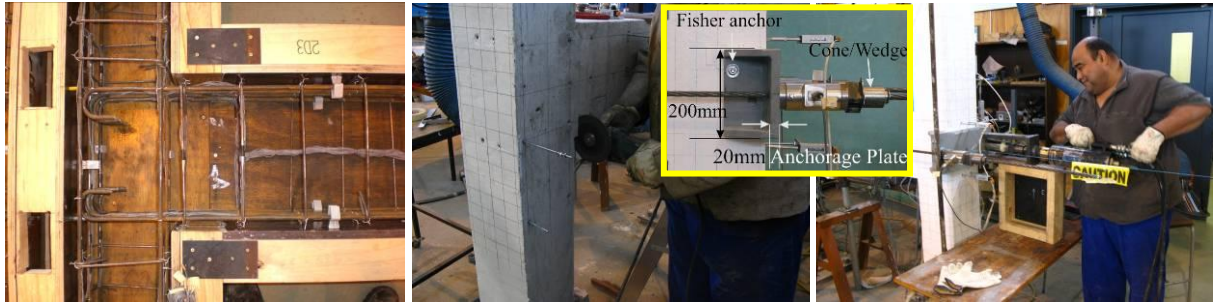


Figure 5: a) BC joint reinforcing details b) Beam weakening -severing beam bottom longitudinal reinforcements c) Applying external post-stressing (insert) anchorage for post-tensioning.

3.2 Experimental Test Setup, Loading Protocol and Instrumentation

To simulate earthquake loading, cyclic quasi-static lateral loading was applied horizontally at the top of the column, as shown in the experimental test setup in Figure 6. The loading protocol used in this experiment consists of two displacement-controlled cycles at increasing amplitudes as follows: 0.1%, 0.2%, 0.5%, 1.0%, 1.5%, 2.0%, 2.5%, 3.0% and 4.0% inter-storey drift, as shown in Figure 6b. Varying axial load of $120kN \pm 4.63V_c$ is implemented, where V_c is the lateral force applied at the top of the column. The varying axial load ratio (4.63) is unusually high, to consider the worst case scenario of an extremely long bay frame, in which exterior columns are likely to be subjected to axial tension force. All the specimens were thoroughly instrumented to measure: a) lateral force applied b) displacement at the top of the column c) local deformation components, and d) strains in the reinforcement. Only selections of the data gathered are presented in this paper due to space constraint.

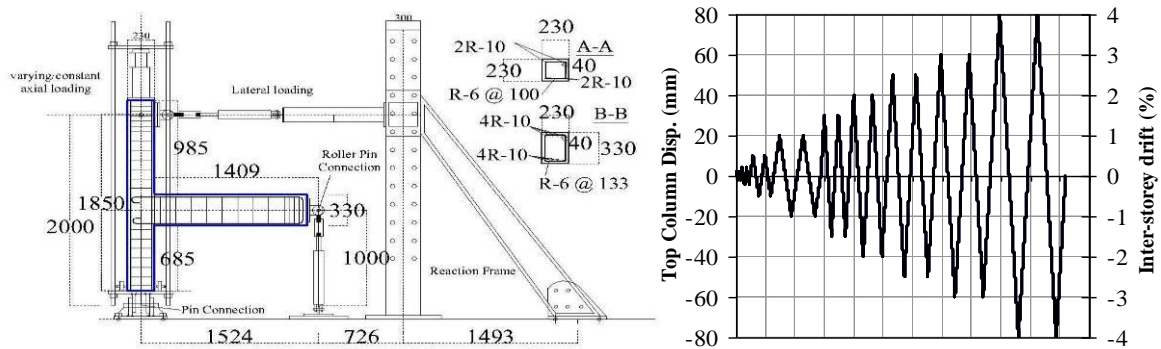


Figure 6: a) Experimental Test Setup b) Loading Protocol

4 RESULTS

The summary of the test results is presented in Table 2 and the hysteretic force-displacement responses of the four bc joints are presented in Figure 7. The cracking and damage patterns at the end of loading of 1.0% and of the final inter-storey drift loading cycles are presented in Figure 9. All bc joints were tested up to 4.0% cycles except for NS-O1 which failed prematurely at the end of the 2nd

cycles at 3.0% lateral drift. Highlights of each specimen's response will be discussed individually.

Table 2: Summary of test results

Test Unit	Failure Mode	Peak Lateral Force (kN)	Inter-storey drift at maximum force, θ (%)	Ultimate inter-storey drift, θ (rad) ¹	$M_{sys-exp}$ ² (kNm)	$M_{sys-cal}$ ³ (kNm)	$M_{sys-exp} / M_{sys-cal}$
NS-O1	Joint Shear Failure	+14.7 -19.4	+1.97 -0.96	+1.0%-II	+12.3 -16.2	+10.4 -15.5	+1.18 -1.05
NS-R1	Beam Flexural, Anchorage	+8.5 -15.1	+0.97 -0.76	-2.5%-II	+7.1 -12.6	+10.4 -15.5	+0.68 -0.81
NS-R2	Beam/Column Hinging	+18.0 -25.8	+1.77 -2.0	-4.0%-II	+15.0 -21.5	+12.5 -21.2	+1.20 -1.01
NS-R3	Beam Flexural Hinging	+17.6 -21.6	± 4.0	- ⁴	+14.7 -18.0	+15.4 -31	+0.95 -0.58

¹ Failure point defined as attained peak force less than 80% of previous peak force; ² Maximum moment in the column

³ Calculated maximum column moment based on Hierarchy of Strength ⁴ No failure (based on the definition) achieved.

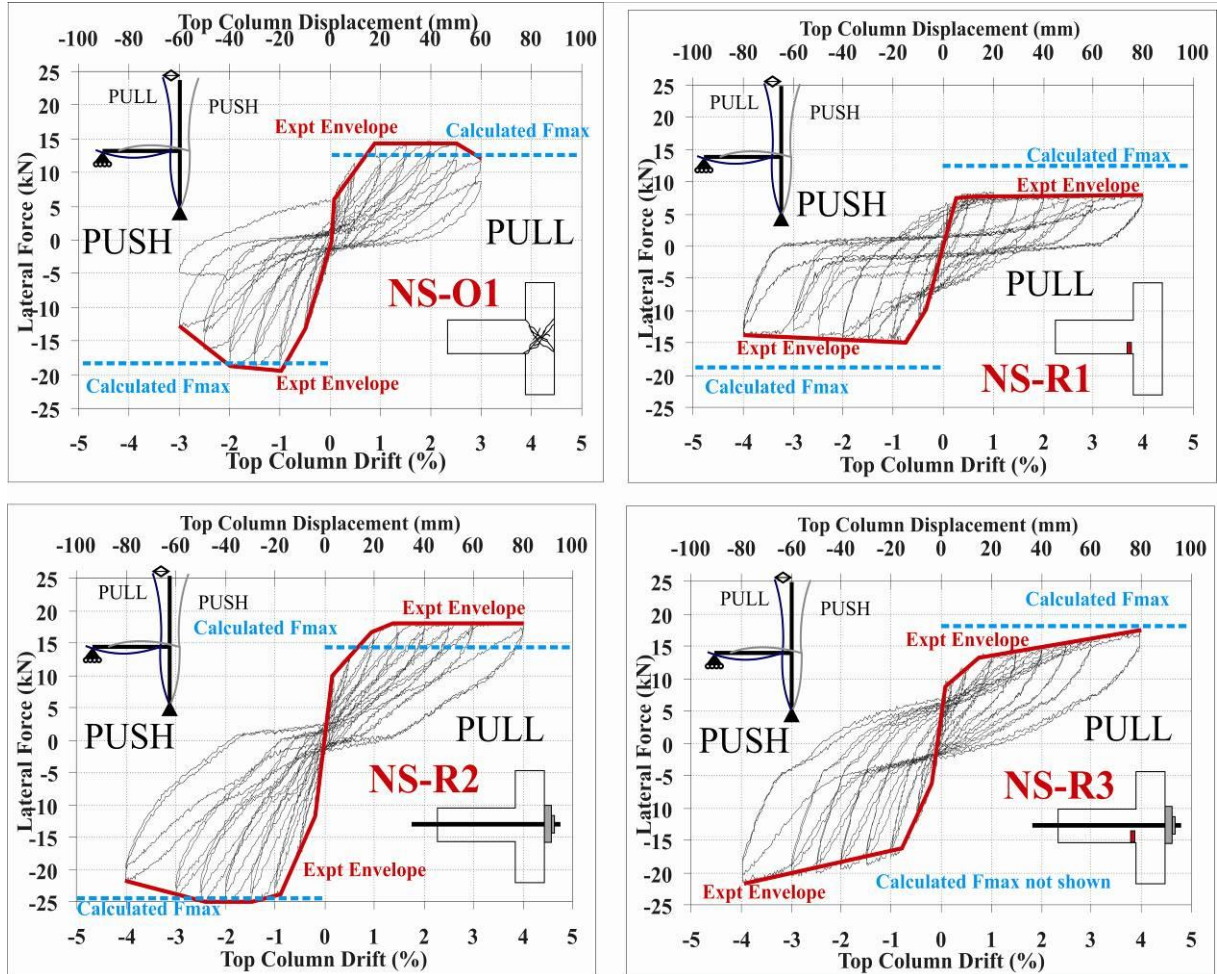


Figure 7: Force-displacement hysteresis curves

4.1 NS-O1 : As-built benchmark bc joint

For the benchmark specimen NS-O1, peak force was attained prior to the joint shear failure (observed as diagonal shear cracking) at the 1st Pull cycle of the 1.0% drift. The joint shear failure leads to the 'ultimate' failure; the peak force during the 2nd cycle was less than 80% of the original peak force. Upon cracking in the joint panel zone, the gradual loss of bond and the push-out force of the standard hook anchorage (see Figure 8b) led to a pinched hysteresis shape, with minimal energy dissipation. During the 1st cycle, pushing to 2.5% drift, the column longitudinal bars began to buckle under the increasing axial load and the load carrying capacity of the bc joint decreased significantly. The concrete wedge failure due to slip/pushout of the hooked end anchorage was further pushed out by buckled column longitudinal bars, as shown in Figure 8a. The failure mode and peak forces were well approximated using the hierarchy of strength and joint principal stresses analysis calculations.

4.2 NS-R1 : Partial SW retrofit – 50% beam positive-flexural weakening

Up to the 2nd Pull cycle at 2.5% lateral drift, stable ‘fat’ hysteresis loop with significant energy dissipation is attained as beam flexural hinging dominates the inelastic mechanism. The discrepancy between the theoretical and experimental maximum forces is possibly due to the bond slip failure along the smooth reinforcing bars, which limits the development of stresses in the reinforcements. As the flexural crack at the weakened section grew, the bond failure, hence slip increased, the hooked end anchorage was forced to act in compression against the concrete cover (Figure 8b). This led to concrete spalling on the joint-column face (See Figure 8a) due to the compression push-out force from the standard hook, thus initiating significant strength and stiffness degradation. Although NS-R1 ultimately failed at Push 2.5% 2nd cycle, this simple retrofit solution has effectively changed the failure mechanism and increased the deformation and energy dissipation capacity of the system, in comparison to NS-O1. It can be seen that up to 1.0% inter-storey drift, no significant damage or crack was observed, where the inelastic mechanism is concentrated at the weakened section. Figure 8c presents a possible upgrade to NS-R1 retrofit that might guarantee better performance.

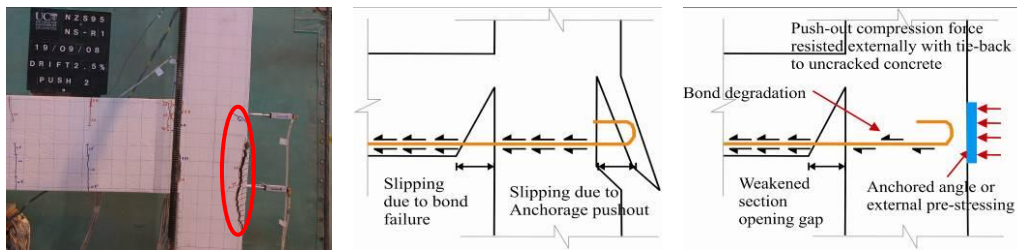


Figure 8: NS-R1: a) Spalling at column-joint face due to push-out force b) Schematic illustration of the bond slip and anchorage push-out failure c) Schematic illustration of possible upgrade to NS-R1

4.3 NS-R2 : External joint pre-stressing/post-tensioning retrofit

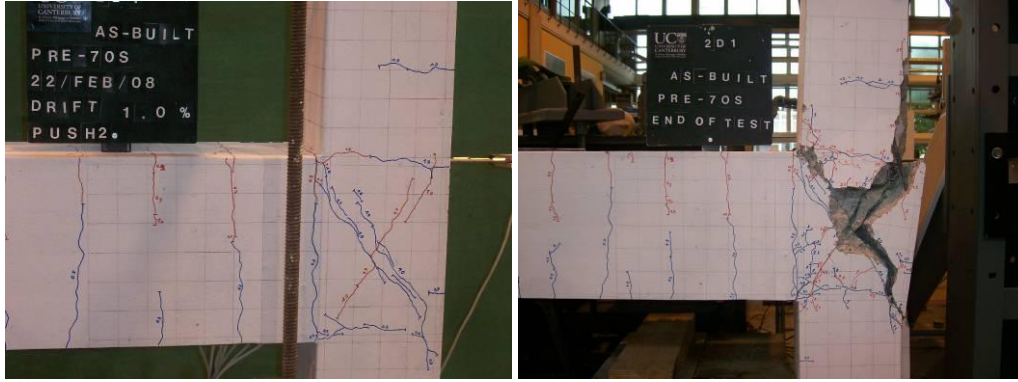
The external joint pre-stressing retrofit was very successful in preventing joint shear failure by increasing the tensile capacity of the joint, as demonstrated in test unit NS-R2. However, with beam-to-column flexural capacities ratio ranging between 1 and 1.8, naturally, strengthening both the joint and beam would lead to column hinging, thus validating the need to weaken the beam in some retrofit scenario. Joint diagonal shear cracks appeared during the peaks of the 1st Pull and Push cycles of the 1.5% drift, as predicted. Premature column hinging suggests the bond failure of the column longitudinal bars. Bond failure and bond-slip limit the column axial-flexural capacity as well as the energy dissipation of the sub-assembly, which is not accounted for in the initial prediction.

4.4 NS-R3 : Full SW retrofit – 50% beam weakening plus external post-tensioning

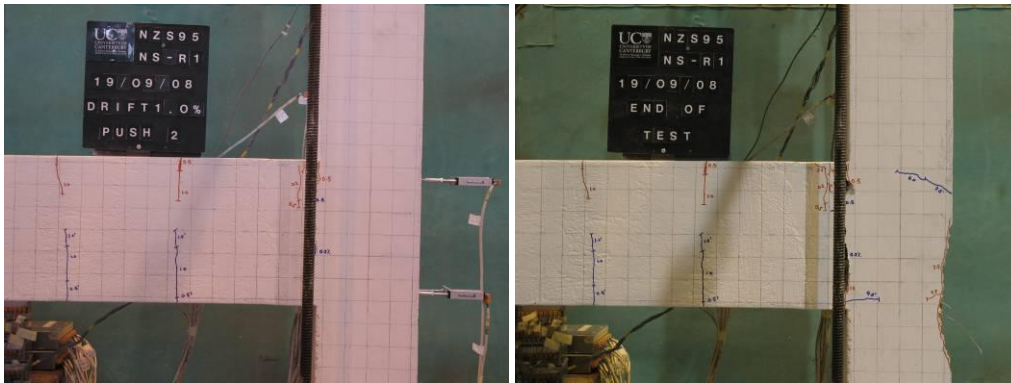
The full SW retrofit test unit, NS-R3, performed very satisfactorily to 4.0% inter-storey drift, without structural failure, strength degradation or signs of loss of vertical load-carrying capacity (e.g. column bars buckling or beam shear). In the Pull direction, stable flexural hinging with considerable energy dissipation capacity was achieved. In the Push direction, minor slipping in the force-displacement curves was observed as bond failure of the plain round bars would still occur. Particularly, stiffness degradation was observed during the 2nd cycles in the Push direction. Some bond splitting cracks were observed in the specimen from a very early stage (Pull 0.5% 1st cycle) (see Figure 9d). Some diagonal cracking is observed along the compression strut within the bc joint, a sign that the principal compression stress might have exceeded the cracking threshold.

5 CONCLUSIONS

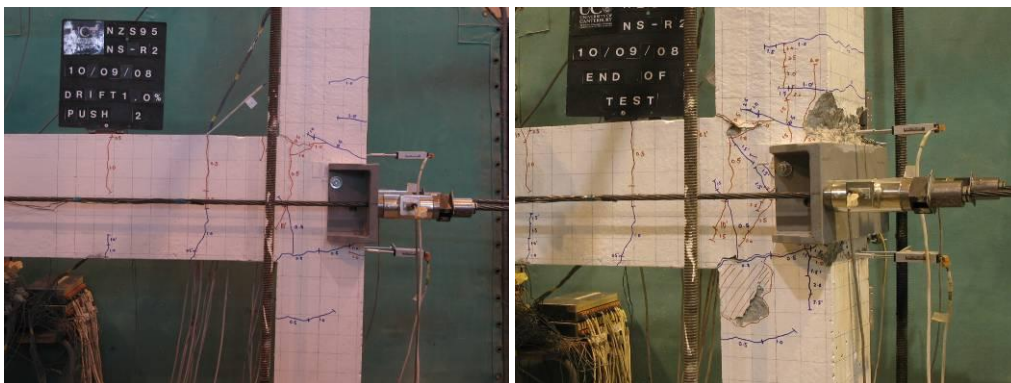
An innovative, counter-intuitive approach for seismic retrofit of rc frames has been presented. By selectively weakening the beam and/or upgrading the bc joint using external pre-stressing, the joint panel zone is protected and an improved inelastic mechanism is activated. In comparison to the benchmark bc joint, NS-O1, an improved performance is achieved in all retrofit solutions. The weakening-only retrofit solution, NS-R1, demonstrated that a reduction of shear force into the joint is



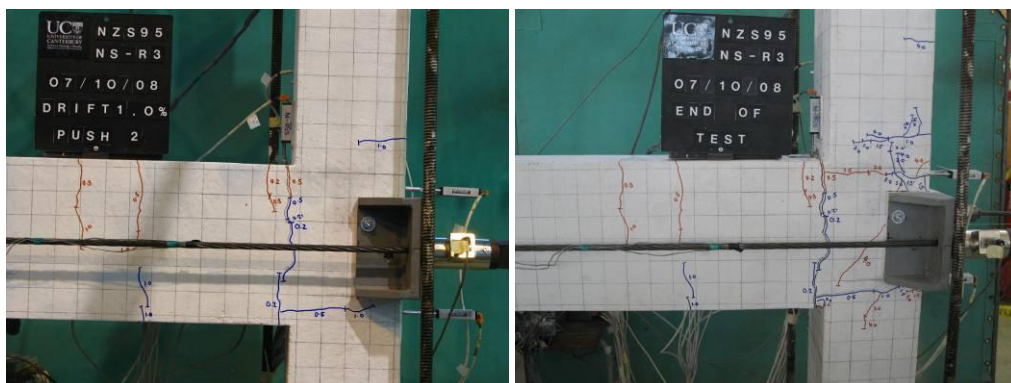
a) NS-O1: As-built benchmark bc joint



b) NS-R1: Partial Selective Weakening retrofit – 50% beam weakening



c) NS-R2: Joint Pre-stressing Retrofit– 120kN external post-tensioning



d) NS-R3: Full Selective Weakening Retrofit – 50% beam weakening + 40kN external post-tensioning

Figure 9: Crack and damage patterns at 1.0% inter-storey drift and at the end of test.

a viable joint retrofit solution, if premature spalling due to hook anchorage push-out is prevented. A post-tensioning-only retrofit as implemented in NS-R2 was effective in protecting the joint if sufficient capacity is available in the column. Lastly, the full SW retrofit implemented in NS-R3 was satisfactory in improving the sub-assembly deformation and energy dissipation capacity. The experimental result presented confirmed the preliminary numerical results published by authors previously. Ongoing research work involves further FEM modelling of the retrofit solutions to investigate effects of anchorage plate, beam weakening cut length, and material properties. Lastly, noting the importance of slabs and transverse beams, and column lap-splice on the overall retrofit performance, five more bc joint sub-assemblies have been recently constructed for testing.

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